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Multiple-support seismic response of Bosporus Suspension Bridge for various random vibration methods

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ABSTRACT

This paper presents a study about the spatial variability effects of ground motions on the dynamic behavior of a suspension bridge by a random vibration based spectral analysis approach and two response spectrum methods. Bosphorus Suspension Bridge built in Turkey and connects Europe to Asia in Istanbul is selected as a numerical example. The spatial variability of ground motions between the support points is taken into account with a coherency function that characterizes the incoherence, wave-passage and site-response effects. Power spectral density function and response spectrum values used in random vibration analyses are determined depending on the recordings of August 17, 1999, Kocaeli, Turkey earthquake. From the results, it can be observed that the structural responses for each random vibration analysis depend largely on the intensity and frequency contents of power spectral density functions.

1. Introduction

Since an earthquake excitation consists of the superposition of a large number of waves with different characteristics, seismic ground motions at the supports can vary significantly along a suspension bridge span. The variations in the support motions can significantly influence the internal forces generated in the structure. So, in calculating the seismic response of suspension bridges, the assumption of uniform ground motion at the supports of this extended structures cannot be considered valid.

In previous studies, analyses of bridges to multiple-support or spatially varying seismic ground motions were performed by various researchers [1–9]. All these studies underline the requirement of the consideration of multiple-support or spatially varying seismic excitations for the dynamic response analysis of suspension bridges. The effect of spatially varying ground motions on the random vibration response of bridges has been studied usually by spectral analysis approach [9–17] and sometimes by response spectrum analysis [8,18,19] in the literature. Comparison of spectral analysis approach and response spectrum analysis of long-span bridges to spatially varying ground motions is meager. Recently, the spatial variability effects of ground motions on the dynamic behavior of deck-type arch and cable-stayed bridges by different random vibration

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methods were investigated by Soyluk and Sıcacık [20]. As known, while suspension bridges cover the center span range of 500–3000 m, cable-stayed bridges cover the center span range of 200–1000 m and steel arch bridges cover the span range of 60–600 m. Random vibration responses of these bridges are different from each other. Although the effect of spatially varying ground motions on the random vibration response of suspension bridges have been investigated either by spectral analysis approach or by response spectrum method, it has not been found any publication in the literature that includes the random vibration based both spectral and response spectrum analyses of suspension bridges subjected to spatially varying ground motions including the incoherence, wave-passage and site-response effects.

The objective of this study is to compare the random vibration response of a suspension bridge to spatially varying ground motions considering the coherency function that characterizes the three spatial variability effects namely the incoherence, wave-passage and site-response. Three different random vibration methods are utilized to determine the dynamic behaviors of the considered suspension bridge, Bosporus Bridge, in this study. As one of these methods is the spectral analysis approach, the other two methods are the response spectrum methods.

2. Formulation

2.1. Spectral analysis approach

Spectral analysis approach is based on the principles of stationary random vibration theory and provides an approximate estimate of the mean of the absolute maximum response of the structure. Any response quantity can be decomposed into dynamic and pseudo-static components, when there is a differential excitation at the supports. The total mean-square response can be obtained from Harichandran and Wang [21]

$$\sigma_z^2 = \sigma_{z_d}^2 + \sigma_{z_s}^2 + 2Cov(z_s, z_d) \tag{1}$$

where $\sigma_{z_s}^2$ and $\sigma_{z_d}^2$ are the pseudo-static and dynamic variances, respectively, and $Cov(z_s, z_d)$ is the covariance between the pseudo-static and dynamic responses. The three components on the right-hand side of Eq. (1) are given by

$$\sigma_{z_s}^2 = \sum_{k=1}^{r}\sum_{l=1}^{r} A_k A_l \int_{-\infty}^{\infty} \frac{1}{\omega^4} G_{\ddot{u}_k \ddot{u}_l}(\omega) d\omega \tag{2}$$

$$\sigma_{z_d}^2 = \sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{r}\sum_{l=1}^{r} \psi_i \psi_j \Gamma_{ki} \Gamma_{lj} \int_{-\infty}^{\infty} H_i(-\omega) H_j(\omega) G_{\ddot{u}_k \ddot{u}_l}(\omega) d\omega \tag{3}$$

$$Cov(z_s, z_d) = \sum_{j=1}^{n}\sum_{k=1}^{r}\sum_{l=1}^{r} \psi_j A_k \Gamma_{lj} \left(-\int_{-\infty}^{\infty} \frac{1}{\omega^2} H_j(\omega) G_{\ddot{u}_k \ddot{u}_l}(\omega) d\omega \right) \tag{4}$$

in which n is the number of modes used in the analysis, r is the number of restrained degrees of freedom, ψ_j is the response z from the jth mode, A_k is the response z due to a unit displacement of support degree of freedom k, Γ_{ki} is the participation factor corresponding to mode i and support degree of freedom k, $H_j(\omega)$ is the modal frequency response function and $G_{\ddot{u}_k \ddot{u}_l}(\omega)$ is the cross spectral density function of accelerations between support degree of freedom k and l.

In the random vibration analysis the mean of the absolute maximum value (μ) can be written as

$$\mu = p\sigma_z \tag{5}$$

where p is a peak factor and σ_z is the standard deviation of the total response [22].

2.2. Response spectrum method

The multiple support response spectrum method based on fundamental principles of stationary random vibration theory was developed by Der Kiureghian and Neuenhofer [22]. This rule provides the response of a linear system subjected to incoherent support excitations directly in terms of the conventional response spectra at the support degrees of freedom and a coherency function describing the spatial variability of the ground motion. The combination rule for the mean of absolute peak response is given in the form [22]

$$E\left[\max|z(t)|\right] = \left[\sum_{k=1}^{m}\sum_{l=1}^{m} a_k a_l \rho_{u_k u_l} u_{k,\max} u_{l,\max}\right.$$

$$+2\sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{j=1}^{n}a_k b_{lj}\rho_{u_k s_{lj}}u_{k,\max}D_l\left(\omega_j,\zeta_j\right)$$

$$\left.+\sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n}b_{ki}b_{lj}\rho_{s_{ki}s_{lj}}D_k\left(\omega_i,\zeta_i\right)D_l\left(\omega_j,\zeta_j\right)\right]^{1/2} \tag{6}$$

in which,

$$a_k = q^T r_k \quad \mathrm{k} = 1, \ldots, \mathrm{m} \tag{7}$$

$$b_{ki} = q^T \phi_i \beta_{ki} \quad k = 1, \ldots, m; \quad i = 1, \ldots, n \tag{8}$$

are the effective influence coefficients and effective modal participation factors, respectively, $u_{k,\max}$ denotes the mean peak ground displacement at support degree of freedom k, $D_k(w_i, \zeta_i)$ denotes the displacement response spectrum ordinate at support degree of freedom k for the frequency and mode of i, and $\rho_{u_k u_l}$, $\rho_{u_k s_{lj}}$, $\rho_{s_{ki} s_{lj}}$ are cross-correlation coefficients between the support motions and the modes of the structure. The cross- correlation coefficients are defined by

$$\rho_{u_k u_l} = \frac{1}{\sigma_{u_k}\sigma_{u_l}}\int_{-\infty}^{\infty} G_{u_k u_l}(\omega)d\omega \tag{9}$$

$$\rho_{u_k s_{lj}} = \frac{1}{\sigma_{u_k}\sigma_{s_{lj}}}\int_{-\infty}^{\infty} H_j(-\omega)G_{u_k \ddot{u}_l}(\omega)d\omega \tag{10}$$

$$\rho_{s_{ki} s_{lj}} = \frac{1}{\sigma_{s_{ki}}\sigma_{s_{lj}}}\int_{-\infty}^{\infty} H_i(\omega)H_j(-\omega)G_{\ddot{u}_k \ddot{u}_l}(\omega)d\omega \tag{11}$$

where $H_i(\omega) = \left[\omega_i^2 - \omega^2 + 2i\zeta_i\omega_i\omega\right]^{-1}$ represent the frequency response function of mode i, and σ_{u_k} and $\sigma_{s_{ki}}$ are the root-mean-squares of the ground displacement $u_k(t)$ and the normalized modal response $s_{ki}(t)$, respectively [8].

2.3. Seismic excitation for random vibration analysis

Both in the spectral analysis approach and response spectrum method the mean of maximum responses depend on the cross-power spectral density function of ground acceleration. This function defined by

$$G_{\ddot{u}_k \ddot{u}_l}(\omega) = \gamma_{kl}(\omega)\left[G_{\ddot{u}_k \ddot{u}_k}(\omega)G_{\ddot{u}_l \ddot{u}_l}(\omega)\right]^{1/2} \tag{12}$$

where $\gamma_{kl}(\omega)$ is the coherency function describing the variability of the ground acceleration processes for support degrees of freedom k and l as a function of frequency ω, $G_{\ddot{u}_k \ddot{u}_k}(\omega)$ and $G_{\ddot{u}_l \ddot{u}_l}(\omega)$ are the auto-power spectral density functions of the ground accelerations for support degrees of freedom k and l.

A general composite model of spatial seismic coherency function proposed by Der Kiureghian [23] in the following form,

$$\gamma_{kl}(\omega) = \gamma_{kl}(\omega)^i\gamma_{kl}(\omega)^w\gamma_{kl}(\omega)^s = \gamma_{kl}(\omega)^i \exp\left[i\left(\theta_{kl}(\omega)^w + \theta_{kl}(\omega)^s\right)\right] \tag{13}$$

where $\gamma_{kl}(\omega)^i$, $\gamma_{kl}(\omega)^w$ and $\gamma_{kl}(\omega)^s$ characterise the incoherence, the wave-passage and the site-response effects, respectively.

For the incoherence effect, resulting from reflections and refractions of waves through the soil during their propagation, the extensively used model proposed by Harichandran and Vanmarcke [24] is considered. This model is based on the analysis of recordings made by the SMART-1 seismograph array in Lotung, Taiwan and defined as

$$\gamma_{kl}(\omega)^i = A \quad \exp\left[-\frac{2d_{kl}}{\alpha\theta(\omega)} \quad (1-A+\alpha A)\right] + (1-A) \quad \exp\left[-\frac{2d_{kl}}{\theta(\omega)} \quad (1-A+\alpha A)\right] \tag{14}$$

$$\theta(\omega) = k\left[1 + \left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-\frac{1}{2}} \tag{15}$$

where d_{kl} is the distance between support points k and l. A, α, k, f_0 and b are model parameters and in this study the values obtained by Harichandran et al. [9] are used ($A = 0.636$, $\alpha = 0.0186$, $k = 31200$, $f_0 = 1.51$ Hz and $b = 2.95$).

The wave-passage effect resulting from the difference in the arrival times of waves at support points is defined as

$$\theta_{kl}(\omega)^{w} = -\frac{\omega d_{kl}^{L}}{v_{app}} \tag{16}$$

where v_{app} is the apparent wave velocity and d_{kl}^{L} is the projection of d_{kl} on the ground surface along the direction of propagation of seismic waves. The apparent wave velocity is considered as 700 m/s for the medium soil type and 1000 m/s for the firm soil type.

The site-response effect due to the differences in the local soil conditions is obtained as [23]

$$\theta_{kl}(\omega)^{s} = \tan^{-1}\frac{Im\,[H_k(\omega)H_l(-\omega)]}{Re\,[H_k(\omega)H_l(-\omega)]} \tag{17}$$

where $H_k(\omega)$ is the local soil frequency response function representing the filtration through soil layers. For the soil frequency response function a model which idealizes the soil layer as a single degree of freedom oscillator of frequency ω_k and damping ratio ζ_k is used as shown below [23]

$$H_k(\omega) = \frac{\omega_k^2 + 2i\zeta_k\omega_k\omega}{\omega_k^2 - \omega^2 + 2i\zeta_k\omega_k\omega} \tag{18}$$

For the spectral analysis approach, the auto-power spectral density function of the ground acceleration characterizing the earthquake process is assumed to be of the following form of filtered white noise (FWN) ground motion model modified by Clough and Penzien [25]

$$G_{\ddot{u}_k\ddot{u}_k}(\omega) = S_0\frac{\omega_g^4 + 4\zeta_g^2\omega_g^2\omega^2}{(\omega_g^2-\omega^2)^2 + 4\zeta_g^2\omega_g^2\omega^2}\,\frac{\omega^4}{(\omega_f^2-\omega^2)^2 + 4\zeta_f^2\omega_f^2\omega^2} \tag{19}$$

where S_0 is the amplitude of the white-noise bedrock acceleration, ω_g and ζ_g are the resonant frequency and damping ratio of the first filter, and ω_f and ζ_f are those of the second filter. In this study, firm and medium soil types are used at the support points of the example bridges and the filter parameters for these soil types proposed by Der Kiureghian and Neuenhofer [22] are utilized (ω_g = 15 rad/s, ζ_g = 0.6, ω_f = 1.5 rad/s, ζ_f = 0.6 for the firm soil type and ω_g = 10 rad/s, ζ_g = 0.4, ω_f = 1.0 rad/s, ζ_f = 0.6 for the medium soil type). As ground motions the recordings of August 17, 1999, Kocaeli Earthquake, Turkey, namely the GBZ000 and ARC000 components recorded at firm and medium soil conditions, respectively, are considered [26]. The amplitude of white-noise bedrock acceleration S_0 is obtained for each soil type by equating the variance of the ground acceleration to the variance of the components of the Kocaeli Earthquake acceleration recorded at firm and medium soil conditions. The calculated intensity parameter values for each soil type are, S_0(firm) = 2.967E–4 m^2/s^3 and S_0(medium) = 2.319E–4 m^2/s^3.

The following relation between the power spectral density of the acceleration and the displacement response spectrum associated with each support motion for the multiple support response spectrum method is derived by Der Kiureghian and Neuenhofer [22]

$$G_{\ddot{u}_k\ddot{u}_k}(\omega) = \frac{\omega^{p+2}}{\omega^p + \omega_f^p}\left(\frac{2\zeta\omega}{\pi} + \frac{4}{\pi\tau}\right)\left[\frac{D_k(\omega,\zeta)}{p_s(\omega)_0}\right]^2 \tag{20}$$

In this expression, p and ω_f are parameters selected by adjusting the power spectral density for low frequencies, τ is the duration of strong motion phase of the motion, ζ is a reference damping ratio, and $p_s(\omega)_0$ is the peak factor of white noise. In this study, the following parameters proposed by Der Kiureghian and Neuenhofer [22] are used: p = 3, ω_f = 0.705 rad/s and $p_s(\omega)_0$ = 2.583.

3. Numerical application

In this study, one of the world's longest modern type suspension bridges, the Bosporus Suspension Bridge is chosen as an example (Fig. 1). The construction of the bridge started in 1973 and completed in 1983. The bridge connecting the Europe and Asia Continents in Istanbul, Turkey is a 1560 m long with a main span of 1074 m and side spans of 231 m and 255 m on the European and the Asian sides, respectively, without any side spans supported by cables. The decks of the side spans at the bridge are supported on the ground by piers. The bridge has flexible steel towers of 165 m high, inclined hungers and a steel box-deck. The horizontal distance between the cables is 28 m and the roadway is 21 m wide, accommodating three lanes each way. The roadway at the mid-span of the bridge is approximately 64 m above the see level. Schematic representation of Bosphorus Suspension Bridge including dimension is given in Fig. 2.

It has been shown that a two-dimensional analysis of the suspension bridge provides natural frequencies and mode shapes which are in close agreement with those obtained by three-dimensional analysis in the vertical direction [27]. For that reason, two-dimensional model of the considered bridge to spatially varying ground motions in the vertical direction for the medium-medium-firm-firm (MMFF) soil condition case is shown in Fig. 3. As the decks, towers and cables are represented by beam elements, the hangers are represented by truss elements. The two-dimensional finite element model

Fig. 1. Bosphorus Suspension Bridge.

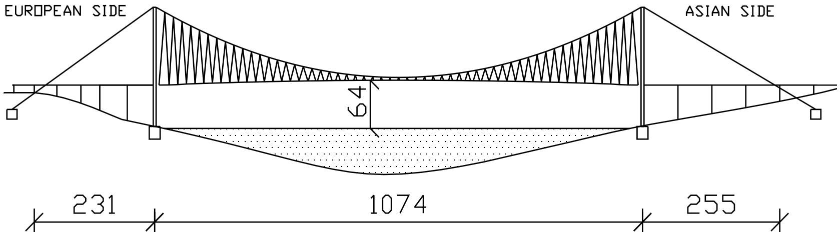

Fig. 2. Schematic representation including dimension (dimensions as m).

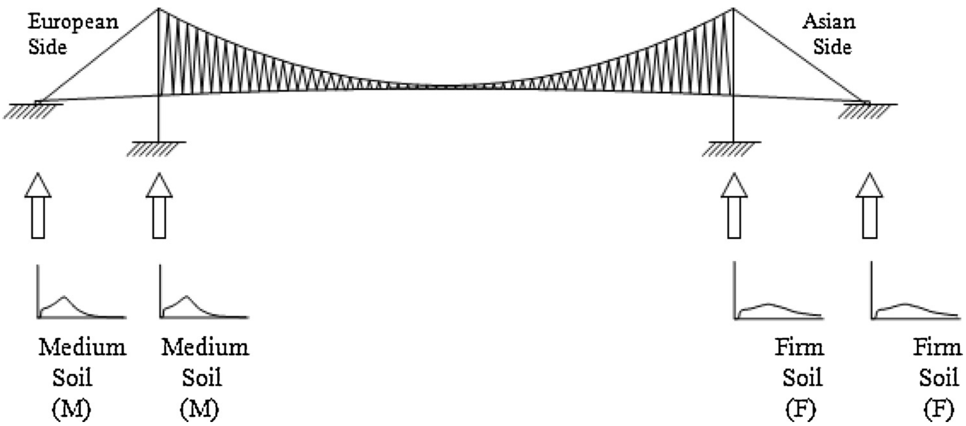

Fig. 3. Two-dimensional finite element model of the Bosporus Suspension Bridge subjected to spatially varying ground motions in the vertical direction for the MMFF soil condition case.
(a) Time history of acceleration.
(b) Acceleration power spectral density function.
(c) Acceleration power spectral density functions for filtered white noise model.

of the Bosporus Bridge with 161 nodal points, 159 beam elements and 118 truss elements is considered for the analyses. The selected finite element model of the bridge is represented by 475° of freedom.

Considered number of modes and natural frequencies of vibration contribute significantly to dynamic response of structures. While, taken into account the first three natural frequencies and modes for usual buildings, the first six natural frequencies and modes for dams and tall buildings are sufficient, more modes and natural frequencies are taken into consideration for complicated structures such as suspension bridges [4]. So, the first 15 modes and a 2% of damping coefficient are adopted for the response calculations. The modal frequencies and periods of the first fifteen modes of the considered suspension bridge model are given in Table 1.

Table 1
The first 15 modal frequencies of the Bosporus Suspension Bridge model.

Mode Number	Frequency (Hz)	Period (sec)
1	0.121	8.286
2	0.161	6.215
3	0.220	4.538
4	0.277	3.617
5	0.365	2.738
6	0.449	2.230
7	0.554	1.805
8	0.574	1.741
9	0.661	1.513
10	0.771	1.297
11	0.896	1.116
12	1.026	0.975
13	1.033	0.968
14	1.036	0.966
15	1.174	0.852

The GBZ000 and ARC000 components recorded at firm and medium soil conditions, respectively, of the Kocaeli Earthquake occurred on August 17, 1999, Kocaeli, Turkey [26] are chosen as ground motions and given in Figs. 4 and 5 since it take placed nearby the bridge site. The components considered are applied to the bridge in the vertical direction with amplitude multiplied by the factor 2/3.

3.1. Random vibration analyses

In this part of the study, the considered suspension bridge model subjected to spatially varying ground motions will be investigated for different random vibration methods. The random vibration methods considered in this study are as follows:

1 Spectral analysis approach of random vibration theory, which considers the Filtered White Noise (FWN) ground motion model as an earthquake ground motion model (Figs. 4c and 5c).
2 Response spectrum method based on the relationship between the power spectral density function (PSDF) and the response spectrum of the input ground motion. The response spectrum displacement is derived from the PSDF of ground acceleration by using Eq. (20). The response spectrum method uses the displacement response spectrum shown in Fig. 6 based on the components of the Kocaeli Earthquake and the PSDF of FWN of firm and medium soil types. This method will be named as PSDF based response spectrum method throughout the study.
3 Multiple support response spectrum method is based on the response spectrum specification of the input motion. This method will be called as response spectrum method in this study. This model uses the displacement response spectrum curves of the two components of the Kocaeli Earthquake as ground motions (Fig. 7).

In this study, it is assumed that while the supports at the European side of considered bridge are founded on medium soil, the Asian side supports are founded on firm soil type. It is also assumed that while the vertical ground motion propagates from the European side to the Asian side. Variable wave velocities are taken into account for the wave-passage effect depending on the local soil profiles. The finite wave velocities are considered as 700 m/s for medium soil type and 1000 m/s for firm soil type. For an earthquake response analysis of many types of structures, the vertical component of ground motion may not be important. For long-span bridges, however, vertical ground motion is important. The considered earthquake ground motions lasting up to 28.00 s are applied to the bridges in the vertical directions, either in power spectral density shape or in response spectrum form. Because of the similarity for the Bosporus Bridge, the Asian side tower responses are not given in this study.

3.2. Displacements and internal forces

The mean of absolute maximum vertical displacements, bending moments and deck shear forces of the deck are compared in Figs. 8–10, respectively for the considered random vibration methods. It can be observed from the figures that the response values calculated by the PSDF based response spectrum method are slightly larger than the values from the spectral analysis, and the response values obtained from the multiple support response spectrum method are generally quite larger than those of the other two methods. While the response values calculated by the spectral analysis are the smallest, the multiple support response spectrum method yields the largest values. At the middle of the deck where the maximum displacements occur, the displacement obtained for the multiple support response spectrum method is 342% and 383% larger than the displacements obtained from the PSDF based response spectrum method and spectral analysis, respectively. The bending moment at the middle of the deck calculated by the multiple support response spectrum method is 316% and 329% larger than those of the PSDF based response spectrum method and spectral analysis, respectively. At the deck point where the shear forces are the

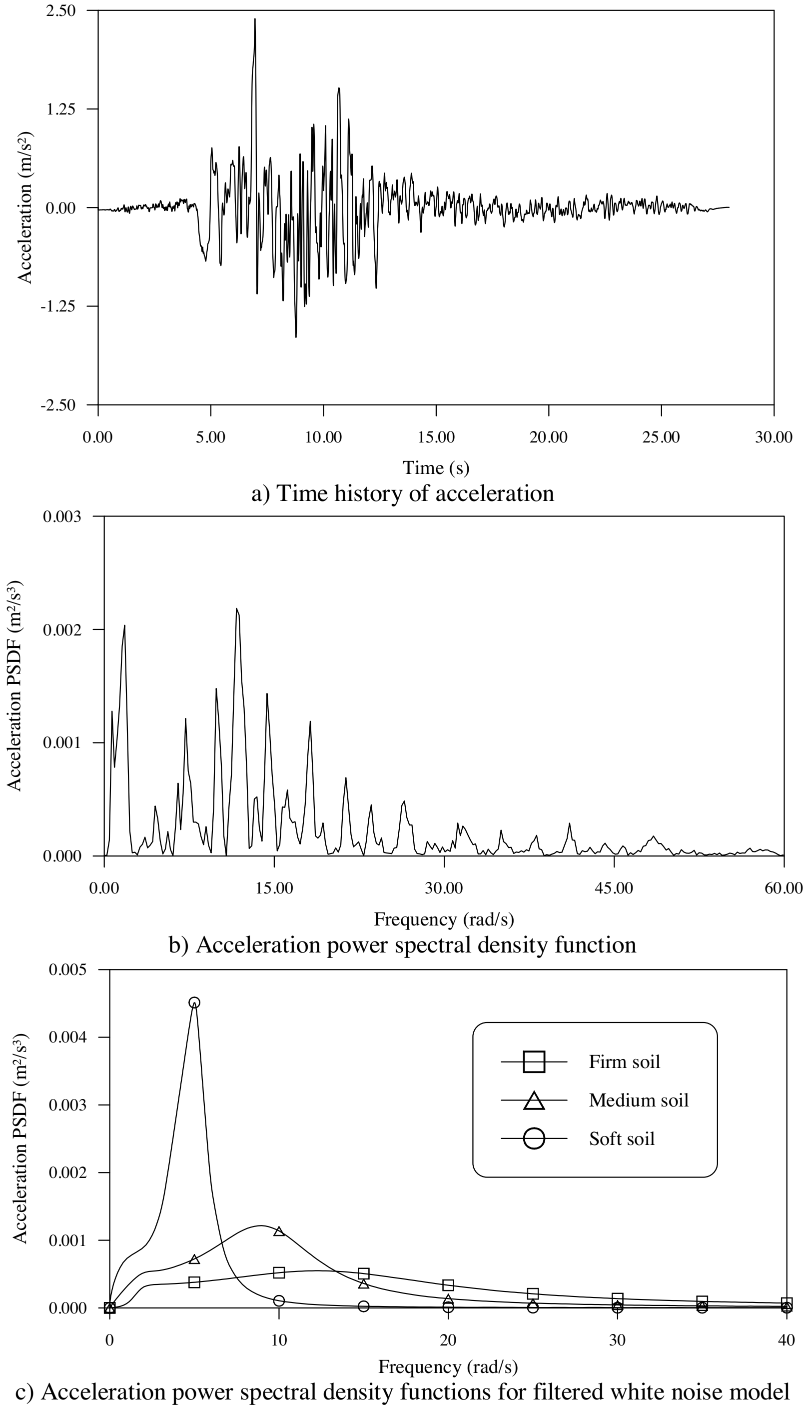

Fig. 4. GBZ000 component of August 17, 1999 Kocaeli Earthquake.
(a) Time history of acceleration.
(b) Acceleration power spectral density function.
(c) Acceleration power spectral density functions for filtered white noise model.

largest, the shear force obtained from the multiple support response spectrum method is 306% and 311% larger than those of the other two random vibration methods.

In Figs. 11–13, the mean of absolute maximum longitudinal displacements, bending moments and shear forces of the tower located at the European side, are also compared for the random vibration methods. It can be seen that the results observed for the deck show themselves again in the comparison of the tower results. The multiple support response spectrum

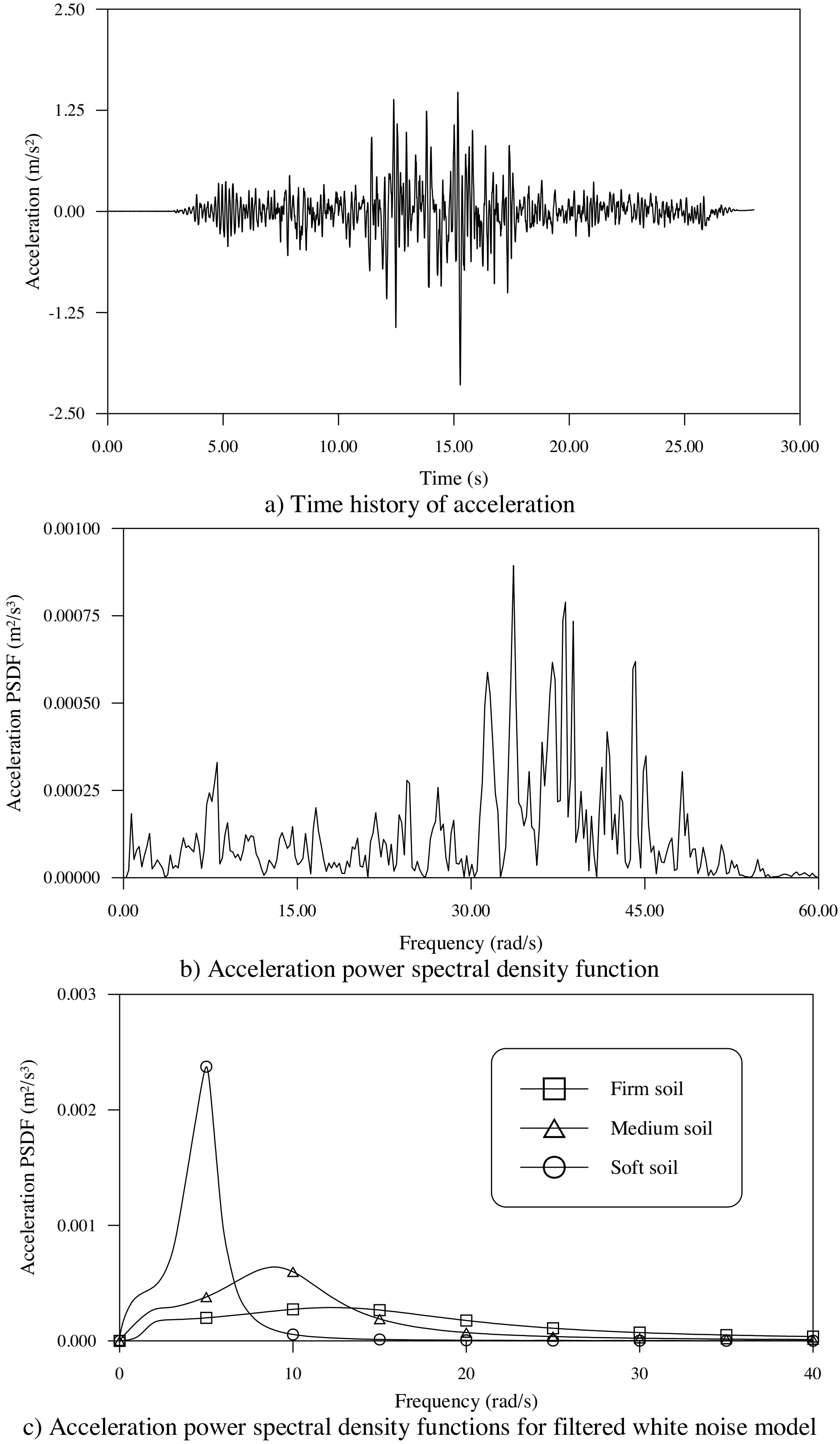

Fig. 5. ARC000 component of August 17, 1999 Kocaeli Earthquake.

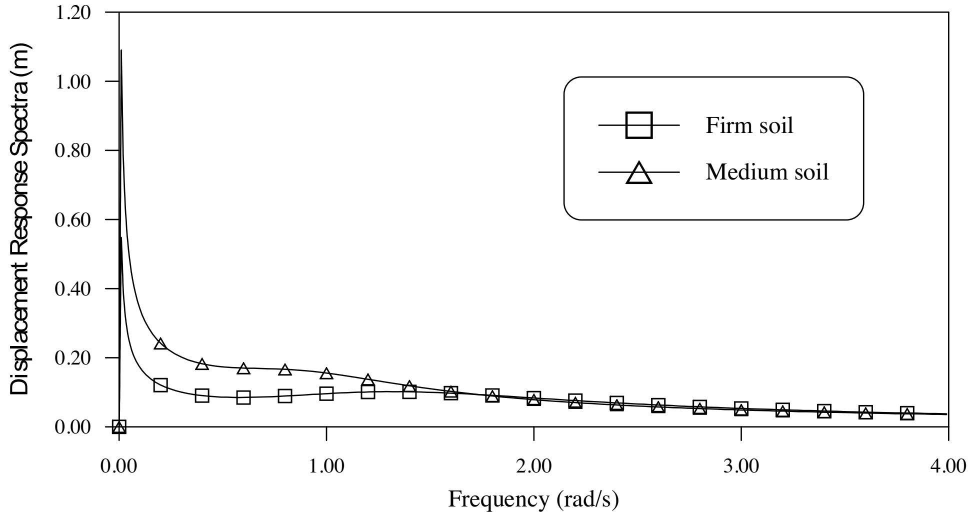

Fig. 6. Displacement response spectra obtained from Eq. (20) for firm and medium soil conditions for $p_s(\omega)_0 = 2.583$.

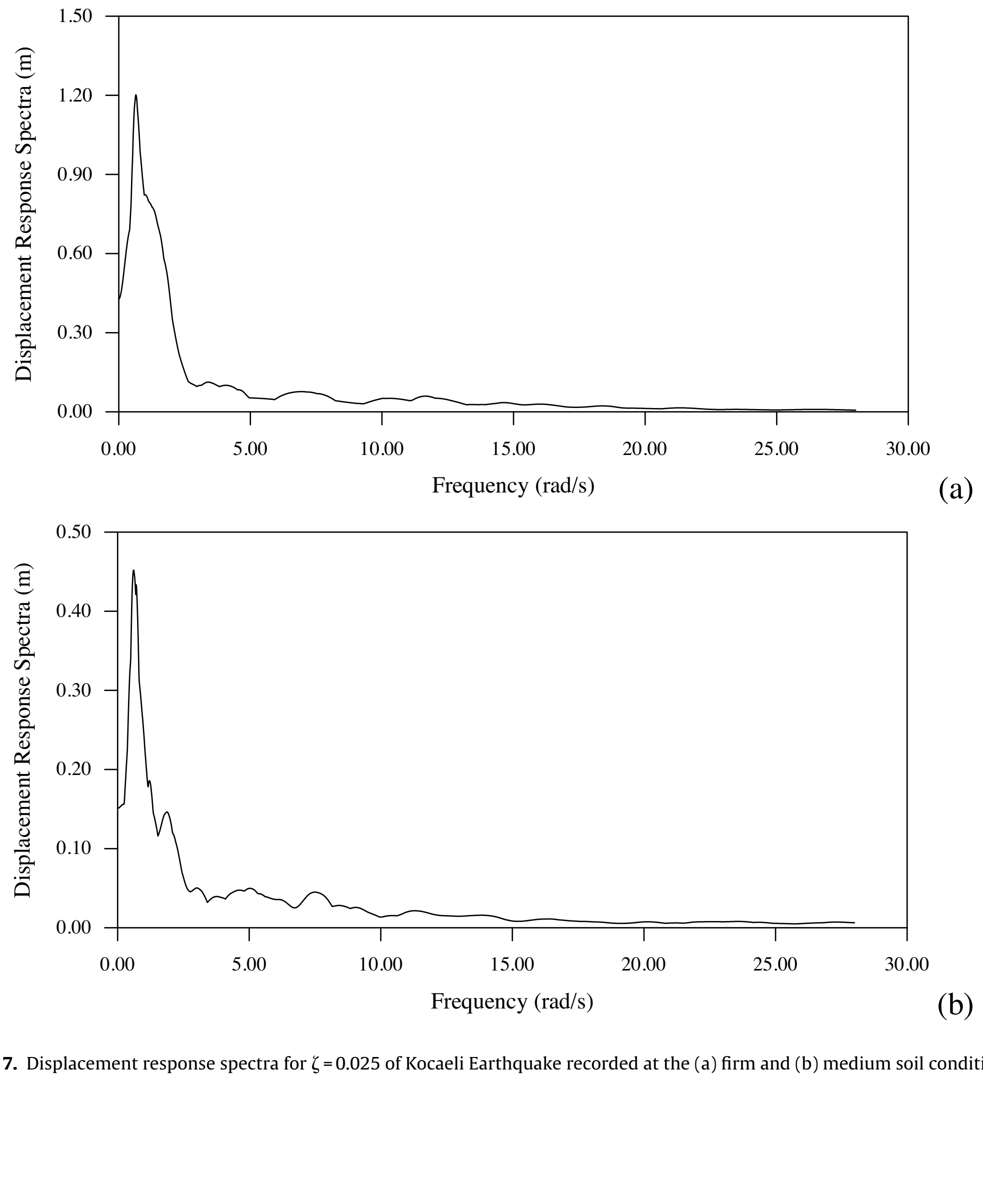

Fig. 7. Displacement response spectra for $\zeta = 0.025$ of Kocaeli Earthquake recorded at the (a) firm and (b) medium soil conditions.

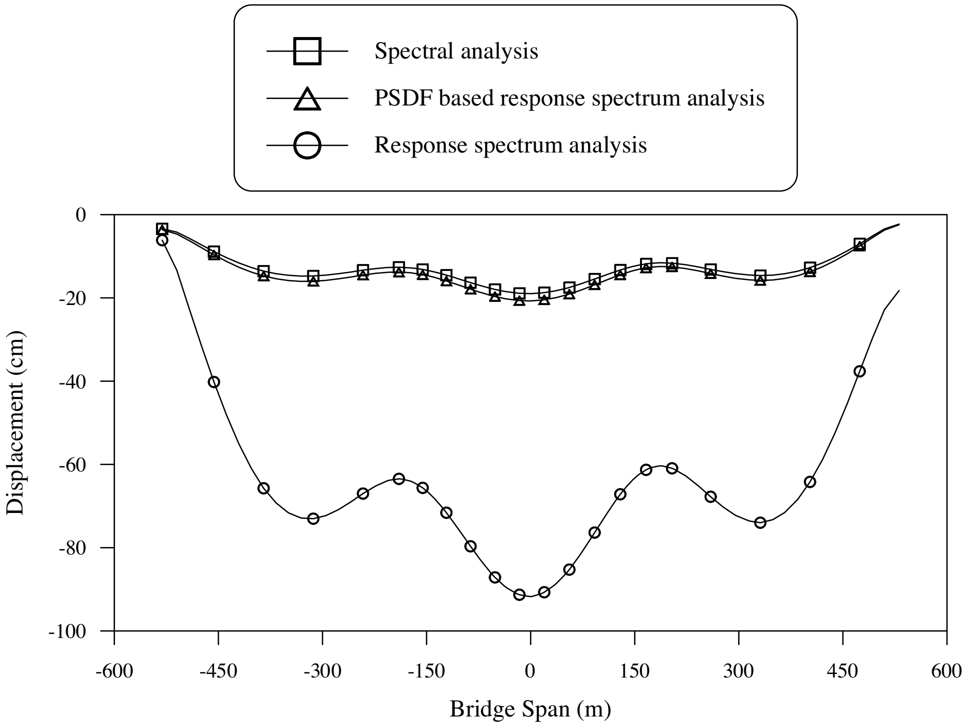

Fig. 8. Mean of absolute maximum vertical displacements of the deck.

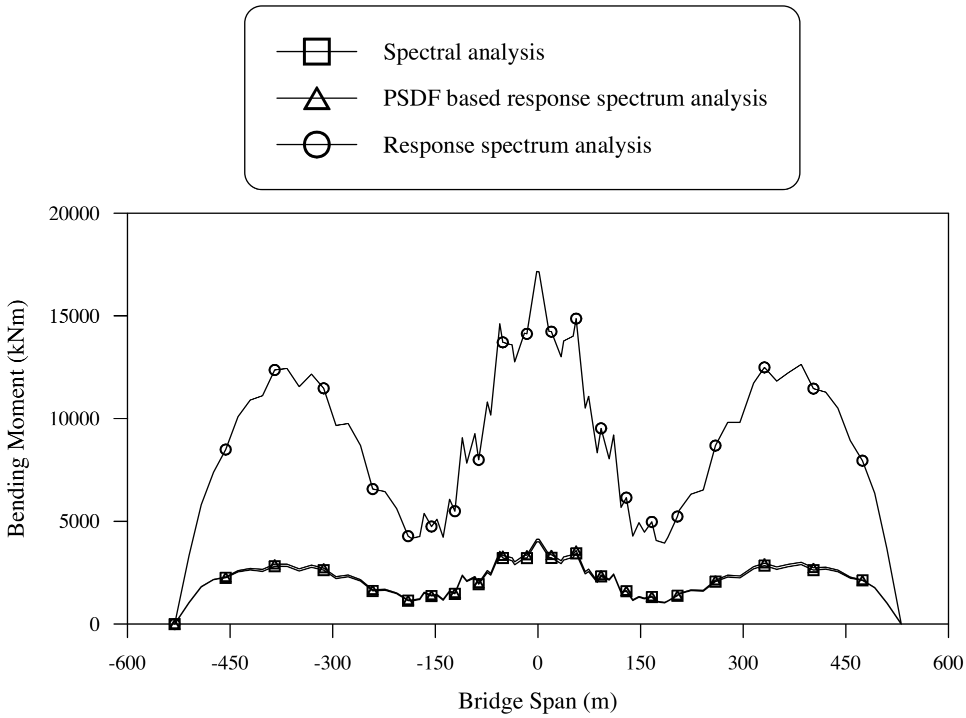

Fig. 9. Mean of absolute maximum bending moments of the deck.

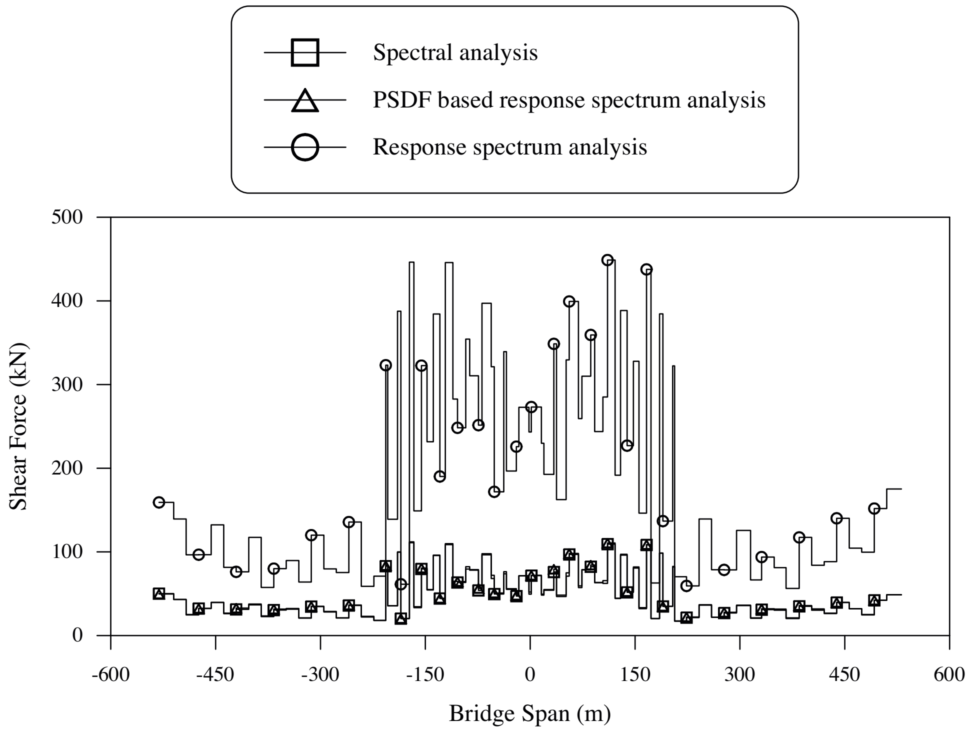

Fig. 10. Mean of absolute maximum shear forces of the deck.

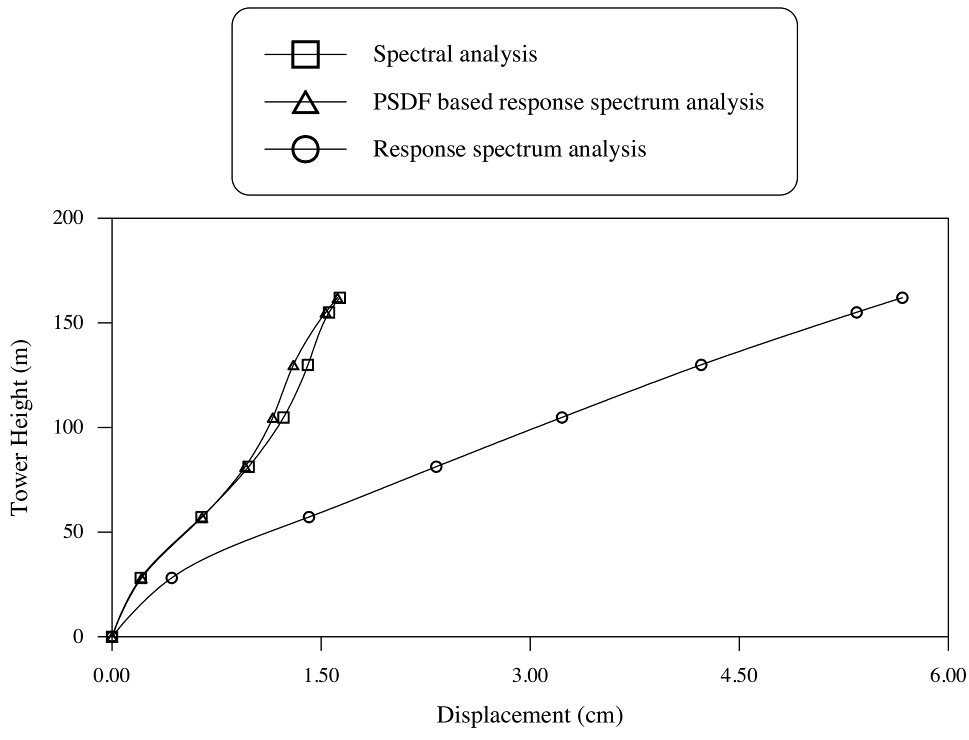

Fig. 11. Mean of absolute maximum longitudinal displacements of the European side tower.

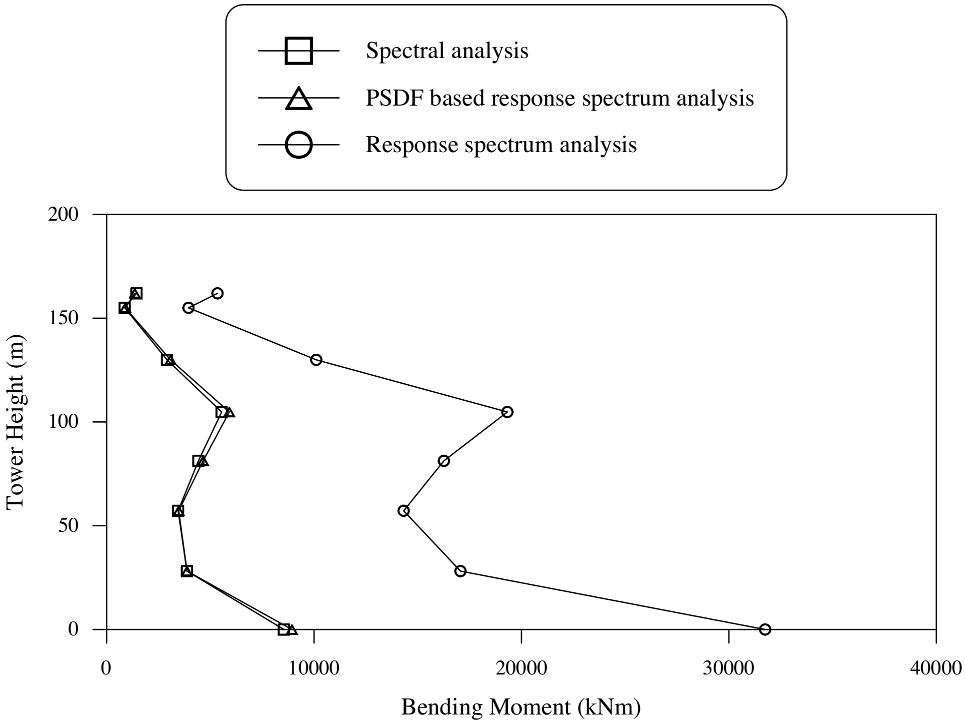

Fig. 12. Mean of absolute maximum bending moments of the European side tower.

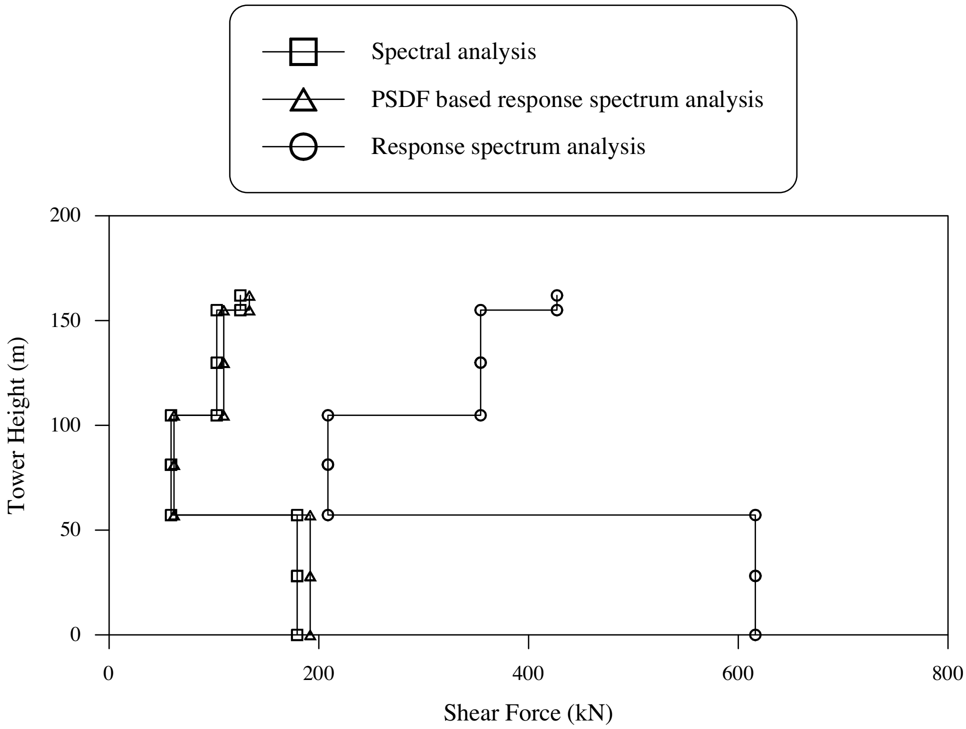

Fig. 13. Mean of absolute maximum shear forces of the European side tower.

method induces 250% and 247% larger displacement value at the top of the tower; induces 256% and 272% larger bending moment value at the base of the tower; induces 222% and 244% larger shear force value at the base of the tower compared to those of the PSDF based response spectrum method and spectral analysis, respectively.

These results outline the fact that the spectral analysis and PSDF based response spectrum method give close response values and the multiple support response spectrum method induces much larger responses. The closeness of the results of the spectral and the PSDF based response spectrum methods can be attributed to the fact that the PSDF based response spectrum method depends heavily on the PSDF of the FWN ground motion model. It should be also highlighted that such a comparison is not entirely fair, because structural responses can show important variations for ground motions even with similar response spectrum. These results indicate that for the suspension bridge model the worst case of random vibration analyses is the multiple support response spectrum method, which results in higher responses depending on the intensity and frequency contents of the PSDF.

The dynamic seismic behavior of deck-type arch and cable-stayed bridges by different random vibration methods are performed by existing study in literature [20]. The variations between the results obtained from various random vibration methods for these type long span bridges are similar to those of the variations for suspension bridge. But the variations for suspension bridge are larger than those of the other two bridge types. Because suspension bridge cover greater span than those of cable-stayed and deck-type arch bridges this results is expected.

4. Conclusions

This study presents a comparative analysis which is performed by a frequency domain spectral analysis approach, a power spectral density function (PSDF) based response spectrum method and a multiple support response spectrum method on an actual suspension bridge model subjected to spatially varying ground motions considering the incoherence, wave-passage and site-response effects. As earthquake ground motions, two different components of August 17, 1999, Kocaeli Earthquake recorded at firm and medium soil conditions are used in this study. The main findings from this study can be categorized as follows:

- The response values calculated by the PSDF based response spectrum method are slightly larger than the values from the spectral analysis, and the response values obtained from the multiple support response spectrum method are generally quite larger than those of the other two methods.
- While the response values calculated by the spectral analysis are the smallest, the multiple support response spectrum method yields the largest values.
- At the middle of the deck where the maximum displacements occur, the displacement obtained for the multiple support response spectrum method is 342% and 383% larger than the displacements obtained from the PSDF based response spectrum method and spectral analysis, respectively.
- The bending moment at the middle of the deck calculated by the multiple support response spectrum method is 316% and 329% larger than those of the PSDF based response spectrum method and spectral analysis, respectively.
- At the deck point where the shear forces are the largest, the shear force obtained from the multiple support response spectrum method is 306% and 311% larger than those of the other two random vibration methods.
- The results observed for the deck show themselves again in the comparison of the tower results.
- The multiple support response spectrum method induces 250% and 247% larger displacement value at the top of the tower; induces 256% and 272% larger bending moment value at the base of the tower; induces 222% and 244% larger shear force value at the base of the tower compared to those of the PSDF based response spectrum method and spectral analysis, respectively.

The responses obtained from the frequency domain spectral analysis are in close agreement with the results obtained from the response spectrum method based on the relationship between the PSDF of the FWN ground motion model and the response spectrum of the earthquake ground motion. Although these two methods are different in character, the agreement of the results can attributed to the fact that the PSDFs for both methods are equal.

The results calculated by the multiple support response spectrum method show a consistent trend with the results obtained from the spectral and PSDF based response spectrum methods; however, there are not close agreement between the multiple support response spectrum method and those of the other two random vibration method results. Reason for the apparent discrepancy between the spectral analysis approach and multiple support response spectrum method is the difference of the variances of ground accelerations described by Eq. (19) for the spectral analysis approach and by Eq. (20) for the multiple support response spectrum method. So, even for the same ground motion, the results obtained for different random vibration methods can vary significantly depending on the intensity and frequency contents of the PSDFs.

Although, the results obtained from this study show the effectiveness of the different random vibration methods on suspension bridges, it is difficult to make general conclusions based on the considered single suspension bridge model and earthquake ground motions because of the complex nature of the problem. To generalize these results, solutions must be obtained using various suspension bridge models and many earthquake inputs.

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